



2.0 Summary of Construction Activities

2.1 Pre-Construction February 2004 through April 2004

The project was officially started in late February 2004. Gila Management, LLC (Tucson, Arizona) was awarded the contract for construction management. Monster Engineering, Inc. (Laporte, Colorado), the Closure Plan engineer, was retained to provide technical support and QA/QC oversight. The project was subdivided into three (3) logical phases:

Phase I – Installation of Dewatering System

Phase II – Removal and Evaporation of Excess Water

Phase III – Final Cover Construction

The first deliverable under the Consent Order was the installation of warning signs around the site perimeter. This activity was completed on March 10, 2004.

The majority of activities in February and March 2004 focused on compiling, and issuing, the bid package for wick drain installation. During this process it was determined that a test program was warranted to determine optimum wick drain spacing and likely consolidation rate. The wick drain test program contract was awarded to Nillex Construction, LLC in mid April 2004, with the provision the full scale installation would also be awarded to them based on the test results.

2.2 Phase I and II, May 2004 through October 2004

Phase I – Installation of Dewatering System

Nillex Construction, LLC mobilized to the site on May 3rd and completed installation of three wick drain test patches the week of May 17th. Concurrent with this activity settlement monuments were set and shallow, lined, ponds were constructed on top of Pond 2 to collect any water extracted by the wick drains. The test program ran from May 21st through June 4th. During this period no measurable consolidation was noted and the wick drains recovered minimal amounts of water, even under assistance from compressed air. After Monster Engineering's review of the data in June, it was concluded wick drains would not be a suitable dewatering method for the Pond 2 tailings. An alternate method, using vertical sumps and horizontal perforated piping, was designed in June. (Refer to Section 3.1 and Appendix J for discussion)

A proposal was solicited from Herm Hughes & Sons, Inc. (Salt Lake City, Utah) to install the revised dewatering system. The contractor mobilized and completed the work in mid July. Additional evaporation ponds were also constructed on top of Pond 2 by a local contractor.

Concurrent with the wick drain testing, the bid package for Phase III, Final Cover Construction, was compiled by Gila Management. In support of this activity, Gila conducted a trade-off study comparing use of locally available clay with Geosynthetic Clay Liner (GCL). GCL was selected for the final cover specification given the known properties, availability and simpler installation.

The Phase III bid package was issued on June 24th and one bid was received from Herm Hughes & Sons on August 4th.

Phase II – Removal and Evaporation of Excess Water

Dewatering of the tailings commenced on July 29th. The process consisted of pumping out each of the dewatering sumps and transferring the water to the closest evaporation pond. The sumps were allowed to recover, and the process repeated as frequently as practical. Due to several years of drought conditions locally, the saturation level in the tailings was low, and the dewatering yield rate was also low. The following table summarizes the estimated quantities of water recovered from late July through the end of September.

Time Period	Gallons in Period	Cumulative Gallons
July 29 through Aug 31	11,250	11,250
September 1 through 30	7,686	18,936

The overall evaporation rate equaled or exceeded the rate the water was recovered. Based on field investigations conducted the week of October 3rd the start of Phase III was scheduled for the third week in October. As of the first week in October the saturation level was observed to be sufficiently below the elevation of the original Pond 2 liner to allow installation of the cover.

Local weather conditions began to deteriorate the week of October 17th. On site precipitation measurements exceeded six (6) inches for the month of October. The abnormal rainfall continued into November, forcing the cancellation of the Phase III contract and an overall revision of the construction schedule.

2.3 Phase IIA, November 2004 through February 2005

A revised, short term, site management plan, designated Phase IIA, was quickly developed in early November to deal with the weather conditions. This plan incorporated the following elements:

- Temporary site grading to improve access and prevent storm water run-off from exiting the footprint of Pond 2.
- Installation of four large (200 ft. x 100 ft. x 3 ft. deep) lined evaporation ponds on top of Pond 2.
- Installation of additional dewatering sumps and perforated drainage piping.
- Daily maintenance and inspection of the site.

In addition to the above, a plan was developed to use portable tanks for temporary storage. One portable tank was rented from Baker Tanks, with the provision for obtaining six (6) more units, however the larger evaporation ponds were quickly put into service, which eliminated the

need for the rented tankage. A contingency plan for off-site disposal of the water was also evaluated, however this option was not pursued because a contract with a suitable waste disposal facility could not be put into place in a timely manner.

A Time and Materials contract was issued to Hughes for the site upgrade work, and lining materials were procured from Western Tank and Lining. The majority of the site upgrade work was completed in January and February 2005.

During January 2005 gas bubbles started to form under the linings of two of the new evaporation ponds. JBR Environmental was contracted to sample the gas and provide recommendations to deal with it. Test results showed the gas was nearly all carbon dioxide. The source is believed to be the reaction of carbonaceous ore disposed of in the impoundment with the low pH water generated by the excess rainfall. The gas bubbles diminished as the pond was dewatered in Phase IIB, and eventually disappeared completely. An engineering review determined that gas venting would not be needed in the final cover. Copies of the JBR report and the evaluation of gas venting requirements are included in Appendix N.

A separate contract was issued to Hughes for site maintenance and dewatering, refer to Section 2.4.

From November 2004 through January 2005 the majority of the effort focused on maintaining containment and control of seepage. Total estimated rainfall at site for 2004 was 17.6 inches, which was approximately double the normal long term average.

2.4 Phase IIB, February 2005 through August 2005

Active dewatering resumed in the latter half of February, and local weather conditions started to trend back towards average by the end of March. The following table summarizes the quantity of water recovered and evaporated during Phase IIB:

Time Period - 2005	Estm'd Gallons in Period - Seepage Collection	Gallons Pumped in Period - Dewatering	Cumulative Gallons to Evap Ponds
Feb. 18 through Feb 28	34,000	28,650	62,650
March 1 through 31	93,500	103,415	328,980
April 1 through 30	68,000	93,350	490,330
May 1 through 31	42,500	162,950	695,780
June 1 through 30	nil	96,150	791,930
July 1 through 31	nil	97,630	889,560
August 1 through 31	nil	70,560	960,140

During Phase IIB project personnel were on site nearly every day to operate the dewatering system and inspect site conditions. Saturation level measurements were started in May and used for on-going analysis of the dewatering rate. Appendix I summarizes this analysis. The normal procedure was to measure the water elevation in all of the sumps, pump them until suction was lost, allow them to recover and repeat the pumping step as many times during day

light as possible. The seepage collection ponds were pumped out as often as necessary to provide maximum available storage capacity.

The Phase III contract was re-bid in May and final commercial terms negotiated with Hughes on August 18th.

A forced evaporation system was designed, tested and set up in May. This system was run continuously during daylight hours until July 17th when the dissolved solids content in the evaporation ponds caused excessive plugging of the spray nozzles.

In early May evidence of a new seepage zone was noted on the southeast face of the Pond 2 embankment. A new lined collection pond was constructed to intercept this seepage. The actual magnitude proved to be minimal and the seep had completely dried up by end of June.

By the end of July analysis of the rate of decline of the saturation level indicated the Phase III work could begin in late August to early September. Provisions were made to continue dewatering, concurrent with Phase III work, as long as possible.

2.5 Phase III, September 2005 through February 2006

Hughes mobilized the first week in September 2005, and started embankment re-grading on September 6th. By the end of the month approximately 90% of the edge of the original Pond 2 liner had been exposed. Embankment re-grading was done with a large excavator working off of the crest of Pond 2, supported by a low ground pressure dozer. Liner edge excavation was done with a mini excavator and hand work. Most of the excess material from embankment re-grading was pushed up onto the top of the impoundment, spread with the dozer and compacted with a sheep foot vibratory roller. The remainder of the material was handled with a loader and off-road haul trucks.

The edge of the existing liner was found in poor condition in some areas. Monster Engineering developed a repair method using GCL material. Refer to Section 3.0 for additional discussion.

The evaporation ponds constructed during Phase IIA were filled in as the embankment re-grading activities progressed around the impoundment. The seepage collection and evaporation system adjacent to the impoundment were removed the week of October 9th. The contents, liners and approximately one foot of the subgrade material were removed and buried in the center of Pond 2. The resultant excavation was inspected for evidence of any evaporation salts. None was noted, and the excavation was backfilled and compacted with clean fill.

Rainy Day Lining, the GCL installation contractor, inspected and accepted the sub-grade on September 30th. Delivery of CETCO GCL materials started the week of October 9th and the installer mobilized and began work on October 24th. Lining and protective cover placement were closely coordinated to comply with CETCO installation guidelines which recommended no liner be left uncovered for more than eight (8) hours. GCL installation was completed on October 30th, and final grading and shaping of the protective cover the first week in November.

All of the borrow material used for the protective layer was obtained from within the Pond 2 fence perimeter. Due to the large quantity of borrow required, the final shape of the diversion ditch had to be revised. Refer to Section 3.0 and Appendix J for details. Final grading of the diversion ditch and other areas outside the Pond 2 footprint were completed in early January 2006.

Erosion protection layer placement commenced shortly after completion of the lining system. The contractor had difficulty obtaining material that met the specified gradation curve. Material was sourced from two different suppliers, and an attempt was made to blend them at site to meet the specification. Based on gradation tests of the blended material, Hecla elected to waive the rigid specification. Refer to Section 3.0 and Appendix F for technical details supporting this decision. The erosion protection layer was completed on December 23rd.

All disturbed areas, except those covered with the erosion protection material, within the fence line, plus the top of Pond 2, were hydro-seeded with a standard Bureau of Land Management recommended reclamation seed mix the week of January 9th. Granite Seed (Lehi, Utah) supplied the following seed mix:

Species	PLS Lbs per Acre
Slender Wheatgrass	9.35
Needle and Thread	0.60
Western Wheatgrass	7.02
Indian Ricegrass	4.38
Bottlebrush Squirreltail	3.65
Total	25.00

Note: PLS = Pure Live Seed

Remaining activities in January and February 2006 consisted of clean-up, resolving punchlist items, equipment demobilization and contract close-out.

2.6 Project Safety Record

The labor hours, for all classifications, expended on the project from February 2004 through February 2006 totaled 10,828. Approximately 66% of this total was construction craft labor. The project experienced no first aid, nor OHSA reportable injuries.

3.0 Summary of Engineering Revisions

3.1 Drainage and Consolidation Method – Vertical Wick Drains

The Final Closure Plan called for the installation of a vertical wick drain system which was to allow for relatively rapid drainage and consolidation of waste materials prior to construction of the final cover system. After drainage and consolidation had taken place, the existing embankment was to be removed, the existing impoundment liner exposed, and the GCL tied into the existing liner. Drainage and consolidation of wastes and temporary cover materials near the embankment were required for:

- Safer and more complete removal of existing unlined embankment materials (those materials higher than the existing liner)
- Stable and relatively firm temporary cover and mixed waste materials at the excavation face
- Improved overall construction conditions for completion of a continuous and uninterrupted tie-in between the new (GCL) liner system and the existing impoundment liner around the impoundment perimeter

Vertical Wick Drain Test Program

In an effort to determine the potential effectiveness of vertical wick drains, and prior to installing the complete vertical wick drain system, a smaller scale test program was implemented. The test program varied conditions at three test pads to determine whether earthen surcharges or introduced air could produce the desired results. The test program included the following:

- Installation of approximately 500 vertical wick drains in three separate test pads located on top of the impoundment near areas of historic embankment seepage
- Application of four feet of earthen surcharge on one test pad
- Application of air pressure through all wick drains in one test pad
- Installation of settlement monuments
- Installation of open stand pipes
- Installation of piezometers
- Excavation of backhoe test pits through the embankment and temporary cover materials near the test pads to observe temporary cover material conditions and phreatic surface levels throughout the test period
- Measurement of temporary cover settlement, wicked fluid, phreatic surfaces, piezometric pressures, and temporary cover stability

Complete details of the field test program are included in Appendix J (Vertical Wick Drain Field Test Program Report, MEI, July 20, 2004). Results from the test program indicated that:

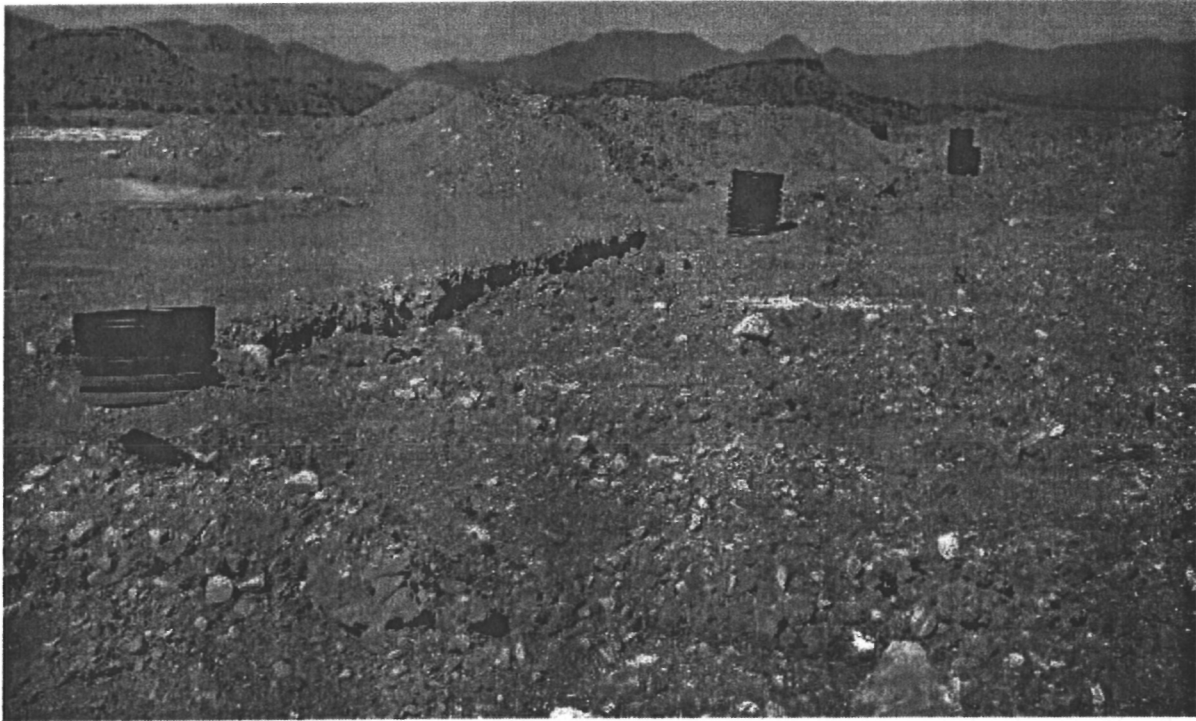
- Vertical wick drains had a very limited capability to drain, consolidate, and stabilize the temporary cover and waste materials in a timely and cost effective manner.
- Significant air pressure and / or a very large surcharge load over an extended period of time would be required to effectively drain and consolidate any of the temporary cover or waste materials.
- Without significant air pressure, wicks by themselves did not remove any measurable liquid from the waste materials.
- With significant air pressure introduced, a very minor amount of liquid was removed, but no measurable consolidation of the cover occurred.

Alternative Drainage / Consolidation System

Based on the test program results and observations, the (then) current conditions of the embankment, temporary cover, and waste materials, the overall objective of the Closure Plan, and results from a small scale temporary evaporation pond constructed on top of the impoundment, it was determined that a more effective, less expensive, and less time consuming method could be utilized to provide for drainage, allow for waste consolidation / stabilization, and intercept liquids migrating towards the impoundment embankment.

The alternative system consisted of installing a system of interconnected vertical sumps and temporary evaporation ponds to quickly lower the phreatic surface and permanently evaporate fluids during the summer months. The system was initially installed where the phreatic surface was slightly higher than the existing impoundment liner. These were locations where temporary collection evaporation ponds had been constructed outside of the impoundment and at other historic seepage locations.

The system included excavation of eight foot deep trenches, approximately 50 feet in from the embankment edge. Large diameter (24") vertical sumps were then placed into the trench every 50 feet and were interconnected by small diameter (4") slotted filter pipe. The small pipe was placed to allow drainage to the vertical sumps. The initial temporary evaporation ponds were shallow (less than 1 foot in depth), lined with 6 to 10 mil black plastic, divided into relatively small cells (20 feet by 20 feet), and constructed on top of the impoundment. Sumps were pumped as frequently as possible into the temporary evaporation pond. The system produced low quantities of liquids, typically between one to two feet of liquid in the bottom of each sump. This fluid needed to be pumped out before more would flow into the sump. Through careful daily management of the sump and pumping system, evaporative potential was maximized. The photograph on the following page shows a series of interconnected vertical sumps on the northeast side of the impoundment after installation.

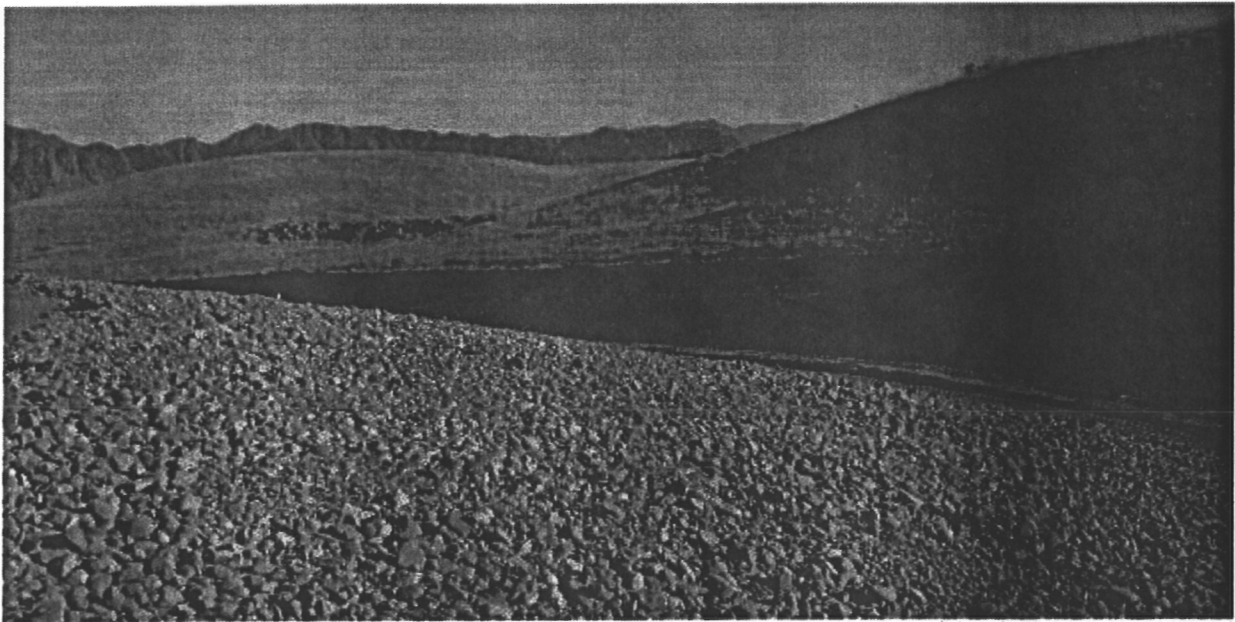


Due to exceptional precipitation which occurred in the fall / winter of 2004 / 2005, the system was eventually expanded with additional sumps, more pumping, and construction of four significantly larger evaporation ponds. This system effectively drained and allowed for consolidation of waste and cover materials near the embankment and in towards the center of the impoundment. It also allowed for safe removal of existing unlined embankment materials, control of remaining temporary cover and mixed waste materials at the excavation face, and adequate conditions for completion of a continuous and uninterrupted tie-in between the new (GCL) liner system and the existing spray-on asphalt fabric impoundment liner.

3.2 Outslope Configuration

The Final Closure Plan called for a final re-graded impoundment outslope of approximately 3.5:1 (h:v). The Plan also called for the existing embankment / waste materials to be cut back to approximately a 1:1 slope prior to installation of the GCL and rebuilding of the embankment. During initial excavation activities it was determined that the exposed temporary existing embankment and waste materials were much looser and less compact than expected. The Contractor therefore reduced this exterior cut slope to approximately 5:1 to 6:1. An as-built plan view and cross-sections are shown in Appendix J. This slope was much more stable and easier to work on both from the equipment standpoint, and for subsequent GCL installation. IN addition, decreasing the outslope also provided for a more stable long-term configuration for the impoundment.

The following photograph shows a typical (and flatter than 3.5:1) outslope near the southeast side of the impoundment.



Overall this modification to the Closure Plan was an improvement in the long-term stability of the impoundment.

3.3 GCL Tie-In

A minor revision was made to the Closure Plan concerning the tie-in between the GCL and the existing liner. The basic design requirements remained the same including:

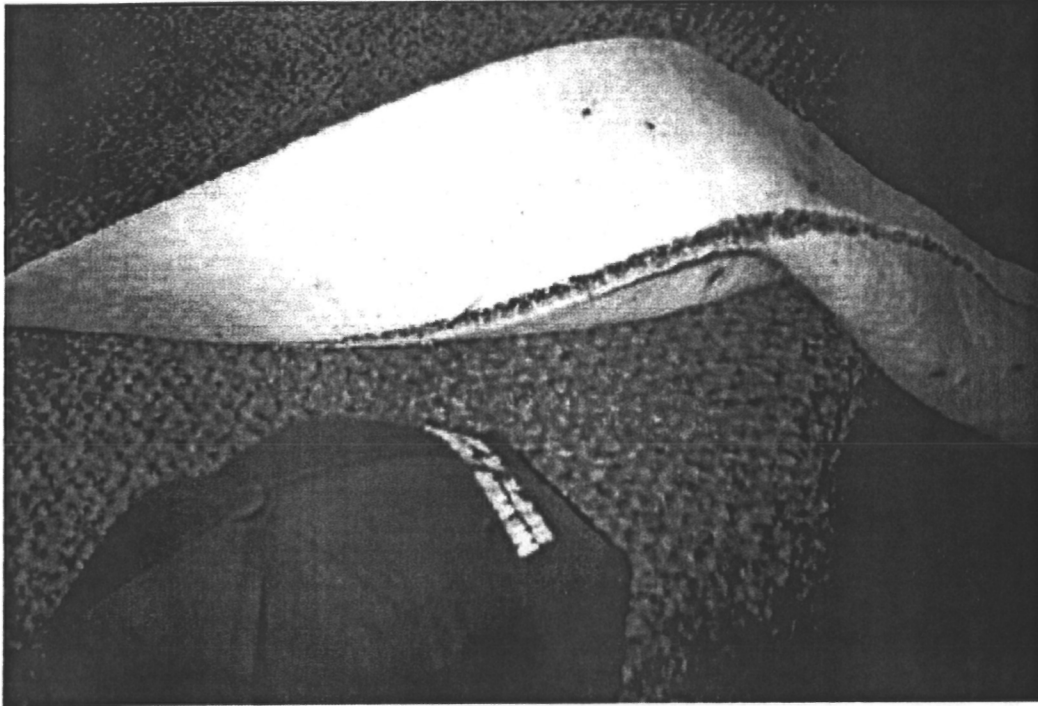
- A minimum one foot horizontal overlap between the new GCL and existing liners
- A small trench on the down slope and inside of the existing impoundment liner to expose the existing liner and allow for temporary storage and removal of minor quantities of liquids which either seeped from the waste materials or collected after precipitation events

Revisions to the Plan consisted of allowing for a much shorter distance between the existing liner and the 18 inch tie-in trench and in areas where needed allowing for sufficient bentonite to be placed and allow for the GCL extend horizontally across the tie-in location between the two liners. The following photograph shows a typical GCL installation on the northwest side of the impoundment. Appendix B contains a figure showing details concerning this revision (Revised GCL to Existing Liner Tie-In Detail and Recommendations after the Preconstruction Meeting – Apex Site, MEI, August 31, 2005).



3.4 GCL Seaming Method

A revision to the GCL to GCL seaming method was approved due to an improvement in CETCO's manufacturing and installation procedures. The Closure Plan had specified that powdered or granular bentonite be hand placed between adjacent and overlapping GCL panels. After the Closure Plan was approved, CETCO developed a "Supergroove" which is now the currently specified method for seams between GCL panels (CETCO 2005). Basically, the Supergroove is not an addition, but a method of pre-cutting the bottom layer of fabric on the GCL. The very small section of fabric is removed to expose the overlying bentonite which exists between the two layers of fabric. This exposed bentonite is then free to form a seal between the upper GCL layer and the lower GCL layer. CETCO has shown that this method produces a seam superior to the older method of hand placing a "bead" of powdered or granular bentonite between adjacent and overlapping GCL panels. The photograph on the following photograph shows the Supergroove. A copy of manufacturer's literature on "Supergroove" is in Appendix C.



3.5 Erosion Protection Material

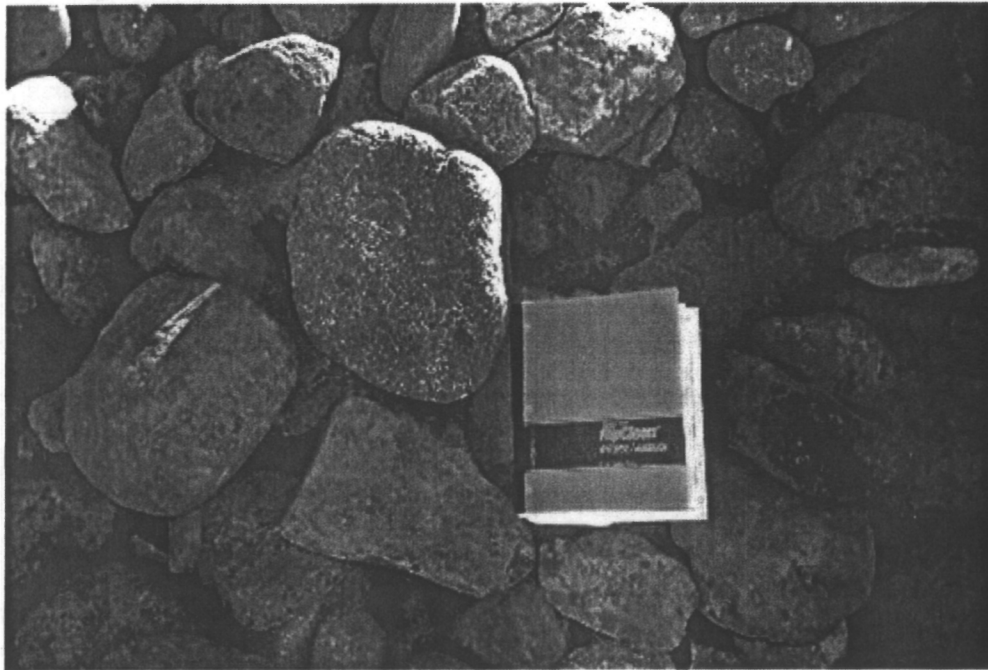
The Closure Plan specified two types of erosion protection materials for the impoundment. The outslopes would be protected from erosion by a $D_{50} = 1$ inch, well-graded rock, and potential migration of the diversion channel would be mitigated by a $D_{50} = 3$ inch, well-graded rock placed on the east side of the impoundment immediately below the $D_{50} = 1$ inch material. The materials were to be well-graded based on a standard engineering design for rip-rap / erosion protection that allows for sufficient interlocking of the materials and prevents raveling or movement of the materials due to force of water and slope on which they are placed.

The Contractor requested and was given approval for an initial revision due to the very limited quantity of $D_{50} = 1$ inch material available in the St. George area. The revision allowed the Contractor to utilize $D_{50} = 3$ inch material for both the outslopes and the diversion channel. As a minimum thickness of 2 times the D_{50} was specified, this revision was basically an improvement in the long-term protection of the impoundment as now 6 inches of rock would be placed on the outslopes instead of the originally specified 2 inches.

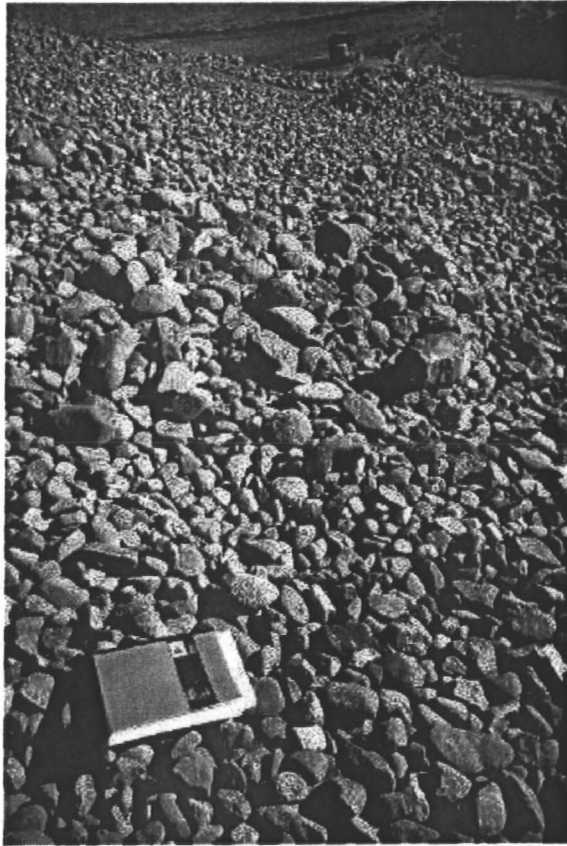
As work progressed in the fall of 2005, the Contractor requested another revision to the specification as they were not able to achieve the gradation specification required in the Closure Plan. The original gradation specification for the $D_{50} = 3$ inch rock is shown in Appendix F as are the results from QA and Contractor provided gradations of materials that were eventually shipped, mixed, and placed on-site. Although the majority of materials originally delivered to the site did not fall exactly within the gradation envelope specified, they were very close. In order to fall within the gradation envelope they required addition of a small quantity of larger (+3 inch and +4 inch materials) and a small quantity of smaller (-1.5 inch materials). The Contractor did have some of these larger and smaller sized materials delivered to the site and

worked at mixing some of them in place with the original materials prior to placement on the outslopes, and mixing some of them after the original materials were placed on the outslopes.

Based on field observations of the final work product (erosion protection materials), and taking into account other revisions to the Closure Plan such as flattening the outslopes from 3.5:1 to approximately 6:1, and increasing the required rock cover thickness from 2 inches to 6 inches, it was determined that the rock materials placed met the intent of the specifications, that is to provide for long-term protection of the outslopes and diversion channel migration. The photograph below shows some of the larger materials delivered prior to mixing.



The photograph on the following page shows some of the materials as they were being mixed on the slope.



The last photograph shows some of the finished product in-place.



3.6 Diversion Channel Final Configuration

The Closure Plan specified that the 1 foot thick layer of Protection Layer materials (those placed directly on the GCL) were to be taken from the borrow area / Diversion Channel located immediately adjacent to and east of the impoundment. Utilization of borrow from this area would also allow for a wider and more gently sloping diversion channel with a normal flow channel located as far as possible from the impoundment. A wide, nearly flat-bottomed channel would provide for much improved long-term erosion protection. Completion of the Protection Layer work required the excavation of approximately 5 additional feet (vertical) from the borrow area, mainly from the southern end of the channel. Because of this additional excavation the overall final slope of the Diversion Channel decreased from the Closure Plan design by approximately 50 percent. This revision will have the effect of decreasing runoff velocities substantially. Much of this additional borrow material was used near the toe of the impoundment outslope to protect the GCL from vehicle traffic as seen in the photograph below.



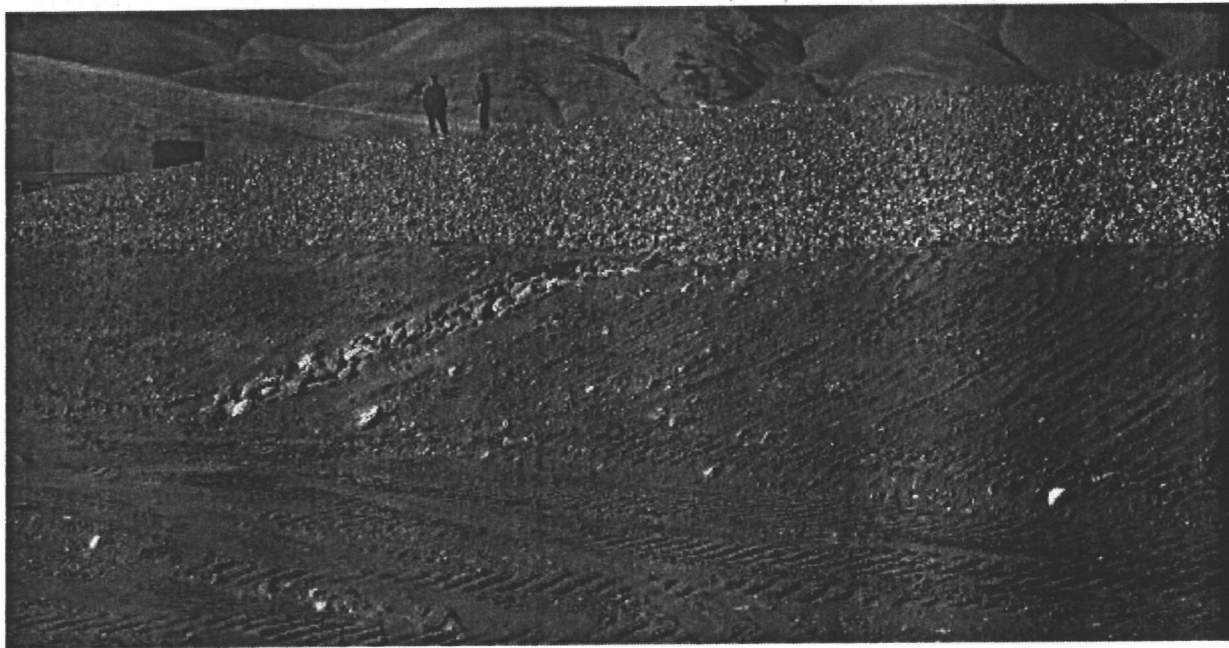
The use of this additional material did provide extra protection for the GCL during construction activities, and in addition, created the much flatter than specified 3.5:1 impoundment outslope. However due to the much deeper Diversion Channel configuration, a revision to the final Diversion Channel slopes and routing of surface runoff was implemented in the field. Drawings and details showing the impoundment to Diversion Channel drainage control features are shown in Appendix J (MEI drawings – December 8, 2005).

This revision allowed for surface runoff from the top of the impoundment (and outslopes) to collect at the bottom of the impoundment outslopes and then flow in a controlled manner through smaller diversion ditches and rock lined diversion swales down to the main Diversion Channel floor. The diversion swales will be monitored for future erosion and potential head-cutting back towards the impoundment toe. The following photos show several different views

of the completed diversion ditch at the bottom of the impoundment outslope and diversion swales. The first photograph shows the diversion ditch along the impoundment toe.



The next photograph shows the completed impoundment outslope and one of the diversion swales that controls runoff from the impoundment toe to the main Diversion Channel floor.



The last photograph shows an overhead view of the completed drainage features on the east side of the impoundment.

